International Conference on Case Histories in Geotechnical Engineering

(2013) - Seventh International Conference on Case Histories in Geotechnical Engineering

01 May 2013, 2:05 pm - 2:20 pm

Observational Method for the Design of a New Ground Improvement Concept Adapted to a Large-Scale Fast-Track Project

Serge Varaksin
ISSMGE, Menard, Inc.

Frédéric Massé
Menard USA

Commemorate the Legacy of Ralph B. Peck
Seventh International Conference on Case Histories in Geotechnical Engineering
April 29 - May 4, 2013 CHICAGO
and Symposium in Honor of Clyde Baker

OBSERVATIONAL METHOD FOR THE DESIGN OF A NEW GROUND IMPROVEMENT CONCEPT ADAPTED TO A LARGE-SCALE FAST-TRACK PROJECT

Serge Varaksin
Chairman TC 211, ISSMGE
E-mail: serge.varaksin@menard-mail.com

Frédéric Massé
Vice-President, MENARD USA
E-mail: frederic.masse@menardusa.com

ABSTRACT

In 2007, the King of Saudi Arabia launched a large scale development program for the construction of a complete university campus on an existing lagoon. The fast-track project was to be completed within a period of 24 months leaving no time for the usual preliminary investigation and design phase. The 5,600,000 m2 site was selected north of Jeddah and a proper soil investigation could not be completed ahead of the start of construction due to difficult site access and challenging existing soft soil condition. In addition to that, because the structural design or even the master plan had not been finalized, the design of the foundation system (loads, footing location...) was going to be completed concurrent with the construction itself. A new concept of foundation support, based on ground improvement, adapted to all potential ground conditions and allowing structures to be randomly located had to be designed and built in record time. The further challenge was to establish the soil parameters and improvement methods. To fit into the extremely tight schedule of the job, the observational method ended up being the best way to define reliable and tested parameters for the ground improvement design and selection to adapt to constantly changing conditions. Late changes in the type of structures combined with difficult site working conditions presented the team with challenges that lead to an innovative use of an optimized combination of Dynamic Compaction, Dynamic Replacement, High Energy Dynamic Replacement and Dynamic Surcharging to meet both the schedule deadlines and the improvement criteria.

1 INTRODUCTION

In early 2007, Saudi ARAMCO was given the task by the King of Saudi Arabia to build the largest university campus in the region – the King Abdullah University of Science and Technology (KAUST). The project covers an area of 36 km^2 comprising academic and administration centres, a residential complex, a research park, a commercial centre, a waste water treatment plant and a marina. This mega project built in a desert environment was scheduled for opening in September 2009 which only allowed 2½ years of construction works. This fast track program coupled with highly variable ground conditions and non-confirmed land-usage due to non-finalisation of the Master Development Plan proved to be a great challenge for the engineers and constructors. Ground improvement works had been included in the site preparation contract to prepare foundations for the low-rise buildings and infrastructures. Because of the many unknowns in terms of geotechnical conditions and structural designs, a formulation of a design concept based on realistic soil parameters on "real-time" adaptation of necessary ground improvement works depending on the prevailing ground conditions and last-minute changes on the intended land-usage by the owner and architects was proposed. The ground improvement works was carried on an area of 2,600,000m^2; all of which needed to be completed in a total period of 8 months.

2 GROUND CONDITIONS

The project site is located about 80 km north of Jeddah in a desert environment. Site investigation was carried out and the results indicated very heterogeneous ground conditions of extremely weak soil deposits. Figure 1 shows the ground conditions covering an area of 1.5 km along the Red Sea. The upper 2 to 5 m and occasionally extending to 9 m below the surface consisted of weak Sabkha deposit. Sabkha is an Arabic expression for "salt flat" to describe recent coastal sediments with high salt content and are characterised by very low bearing capacities and low N_{SPT} values. Generally, sabkha consist of sand deposit mixed with silt and clay and it behaves close to a liquid state. The water content ranged between 35% and 48% with fine contents (size < 63 μm) between 28% and 56%. Cone resistance q_c from CPT (cone penetration test) varied from 0 to 0.2 MPa and N_{SPT} values varied from 0 to 2 blows suggesting a very weak soil. The surface elevation was around RL+4.0m with the ground water table at around RL+0.5m. Figure 2 shows the weak bearing condition of the ground.

Figure 2 Low bearing capacity of Sabkha soil

3 PRE-TREATMENT IN-SITU TESTS

Pre-treatment in-situ tests consisting of 672 CPT and 2,430 PMT (pressuremeter test) were performed across the site. From these tests, the heterogeneity of the underlying soil was confirmed. Indeed, many CPT tests located at distances of 30m from each other showed huge differences as shown in Figure 3.

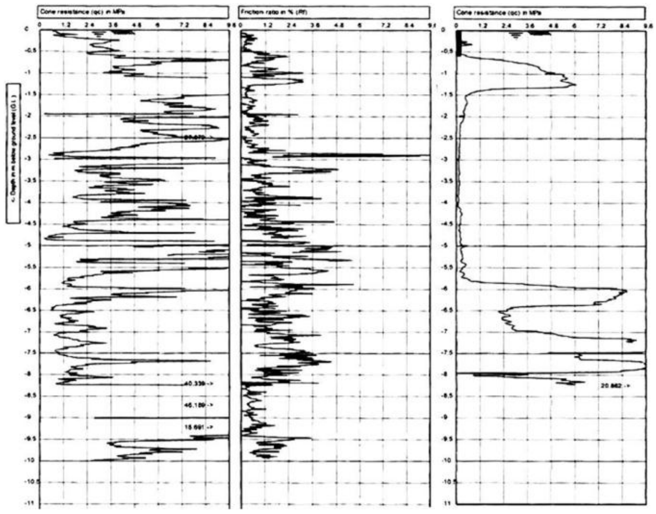
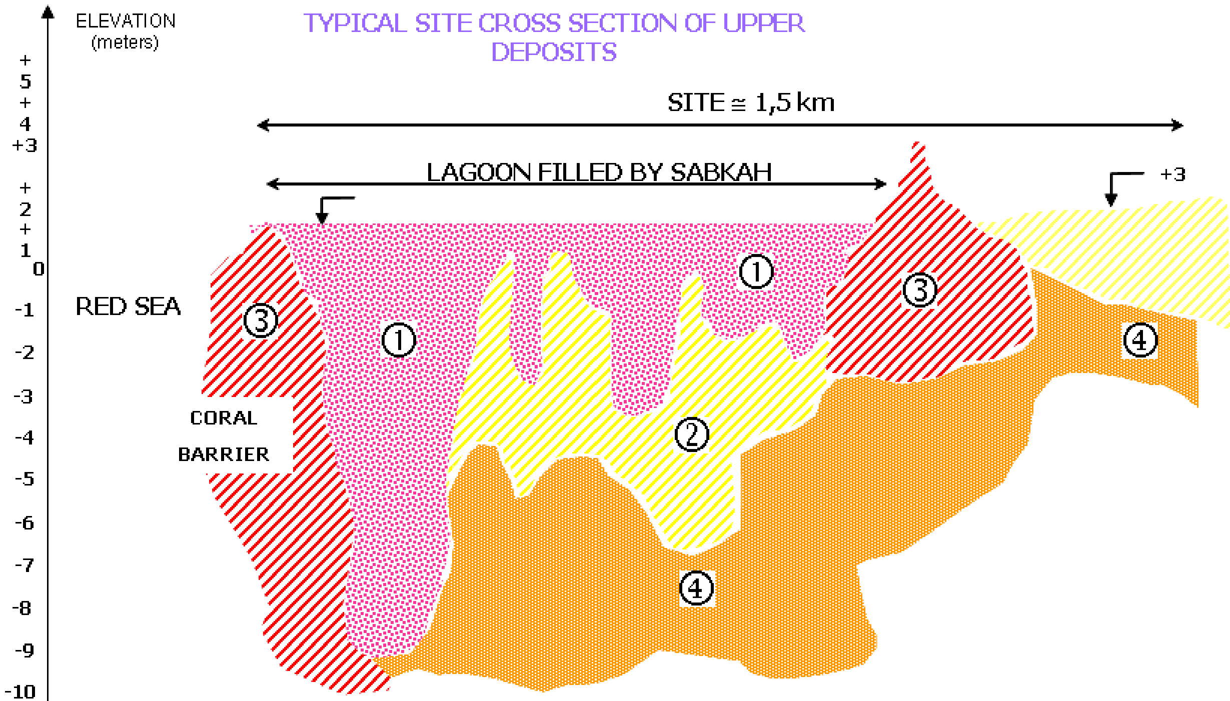


Figure 3 CPT results located 30m from each other

Under such highly varied ground conditions, it was impossible to define a single ground improvement method to treat the whole 2,600,000 m^2 area. A combination of methods was deemed necessary.



LAYER	USC	w %	%fines	N	Qc BAR	Fr %	PL BAR	Ep BAR
1 - SABKAH	SM + ML	35-48	28-56	0-2	0-2	1,2-4	0,4-1,9	[illegible]-17
2 - LOOSE SILTY SAND	SM	-	15-28	3-9	12-45	0,5-1,2	2,1-4	18-35
3 - CORAL	-	26-35	-	6-12	-	-	5,1-7,2	35-60
4 - LOOSE TO MED DENSE SAND	SM	-	12-37	3-18	15-80	0,5-1,8	4-12	28-85

Figure 1 Ground condition across the site

4 PROJECT CONSTRAINTS

With a short construction period of 2½ years, the project was fast-tracked with project planning, design and construction being conducted in parallel and in tandem. At the start of the works for the site preparation phase (which included the ground improvement works), the Master Development Plan was not ready. Hence, the ground improvement was in fact needed everywhere since the locations of buildings, structures and infrastructures were not defined yet. Furthermore, the construction of buildings, structures and infrastructures was scheduled to commence 6 months after the commencement of ground improvement works while thetotal duration of the ground improvement work was going to be around 8 months.The presence of the highly heterogeneous weak soil deposits across the site posed further complications. Already without knowing the precise locations of the treatment areas and given the fact that future constructions may take place anywhere in this huge site when the Final Master Development Plan becomes available at a later date, a design concept for the ground improvement works capable of being adaptable to this complex situation was needed. Several meetings with the client, architects and engineers led to a workable concept that defined boundary conditions for an economic and fast-track construction schedule. The following typical design and performance criteria were defined (Figure 5):

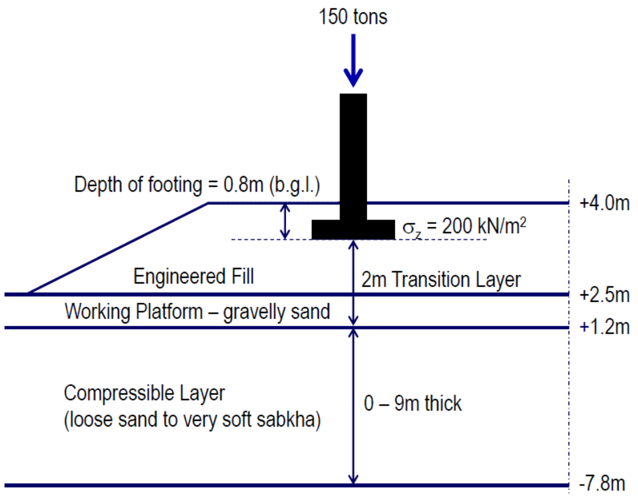


Figure 5 Design concept for ground improvement works

- Footing loads not exceeding 1,500kN (150 tons).
- Net allowable bearing capacity of 200 kN/m².
- Maximum footing settlement not more than 25mm.
- Maximum differential settlement not exceeding 1/500.
- Based on the above criteria, the locations of footing remained undefined and they may be located anywhere on site after the ground improvement works. The ground improvement contract overall schedule was a total of 8 months with interim handover of improved areas after 6 months for the start of th construction of buildings and infrastructures.

Based on this design concept, a formulation of the most suitable ground improvement methods was evaluated. Due to the heterogeneous ground conditions, a combination of 3 methods was selected as follow:

- Dynamic compaction (DC) in the sandy deposits.
- Dynamic replacement (DR) in the Sabkha deposits.
- Heavy dynamic replacement (HDR) with sandy gravel columns of 2.5m diameter up to 5m depth with 3 m surcharge in the Sabkha deposit exceeding 5 m depth.

Figure 4 shows the method selection chart and procedures.

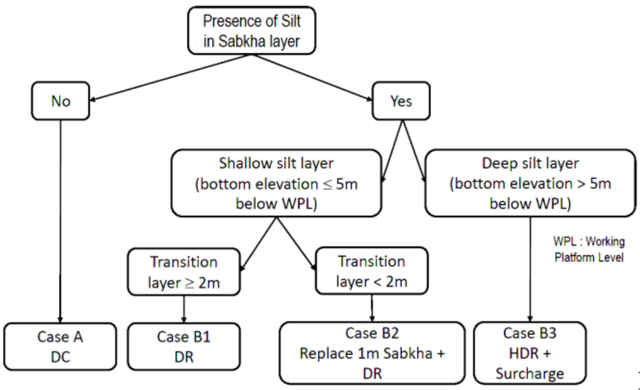


Figure 4 decision chart for methods selection and procedures

Observational method:

During DC and DR works, the heterogeneous ground conditions were obviously reflected by the penetration of the pounder. In one case, with a 400 ton.m (i.e. 20 tons dropped from 20 m) impact the penetration of the pounder was only about 15 cm and less than 25 m away, the penetration of pounder could be 150cm with a 200 ton.m impact so 10 times the penetration with half the energy. This is a factor of more than 10 and these observations allowed to differentiate quadrants of 100 m2 for DC or DR methods.

Figure 8 shows the design minimum requirements to achieve the above mentionned performance criteria and to allow random construction of footings on improved ground.

- The presence of a working platform (gravelly sand) at least 1m thick for stability of construction plant and equipment.
- The presence of an engineered fill layer at least 2m thick below the footings to act as a load distribution layer (arching transition layer) above the composite soil-DR column layer (load transfer platform).

5 ORGANIZATION OF WORKS

With unknown locations of buildings and infrastructures, a highly variable ground conditions coupled with a fast track program, the only applicable and practical way to differentiate areas of treatment was to use the Observational Method based on visual inspection of the real-time response and measured behaviour of the soil subjected to the treatment works being performed. With the formulation of a design concept based on realistic soil parameters,a thorough site quality control program coupled with real-time analysis of measured field parameters to satisfy the performance criteria, the ground improvement work was adapted on site based on the directly-observed prevailing ground conditions given by site parameters such as the penetration of the pounder under each impact. This information was used to determine the type of improvement of either DC, DR or HDR and the required compaction energy used. Based on a "proof" impact grid of 5.5m by 5.5m,this method allowed to create a global site map of the various treatment areas requiring DC, DR or HDR and its variable compaction energies.

This real-time monitoring necessary to cover to the 2,600,000 m2 of treatement area with such variable soil conditions required a large workforce of skilled technicians and operators. Experienced geotechnical engineers were assigned to the real-time mapping of every impact prints. During the peak production, the project team of 90 staff consisted of 10 people from the management team, 32 persons for the production team, 18 workers for the plant and mechanical team, 16 persons in the survey team, 8 technicians from the geotechnical team, 6 specialists from the site safety team. A total of 13 units of DC/DR rigs with capacity of 18 to 25 ton line-pull, 3 CPT rigs, 3 Pressuremeter PMT rigs and 1 SPT rig were deployed for the works. Figure 6 shows the DC/DR rigs mobilized for the project.

Figure 6 DC/DR rigs

6 QUALITY CONTROL

Field observation remained as the primary control for the proper execution of the ground improvement works. Some of the pertinent visual indications are given in Table 1. This was supplemented by CPT, PMT and SPT testing. A total of 76 test pits with soil sampling and 462 nos. of grain size analysis were performed during the works.

Table 1 Visual control of operation parameters

	DC	DR / HDR
Type of impacts	High intensity / max. energy	Low intensity / low energy in first 2 blows
Type of pounders	4 m² (15 – 23 tons)	3 m² (variable weight)
Drop height	Typical 20 m	Adapted to heave intensity (from 5m to 20m)
Heave	Negligible	High decreasing with increasing passes
Diameter of prints	3.5 – 4 m	2.3 – 3.5 m
Penetration	~ 25 cm / blow	~ 100 cm / blow
Water observed	frequent	rare
Rest period between phases	1 – 3 days	7 – 21 days
Transition layer	Not required	Required for arching effect
Surcharge (preloading)	Not required	Required for HDR (case B3)

6.1 *Quality Control for DC – Analytical method*

The quality control for DC is based on the design rules given in the pressuremeter manual D-60AN (Sols-Soils No. 26, 1975). The calculations for bearing capacity and settlement are based on the limit pressure (P_L) and pressuremeter modulus (E_M) obtained from PMT tests. More than 2,000 PMT test locations were carried out. A typical PMT results for DC treatment is given in Figure 7.

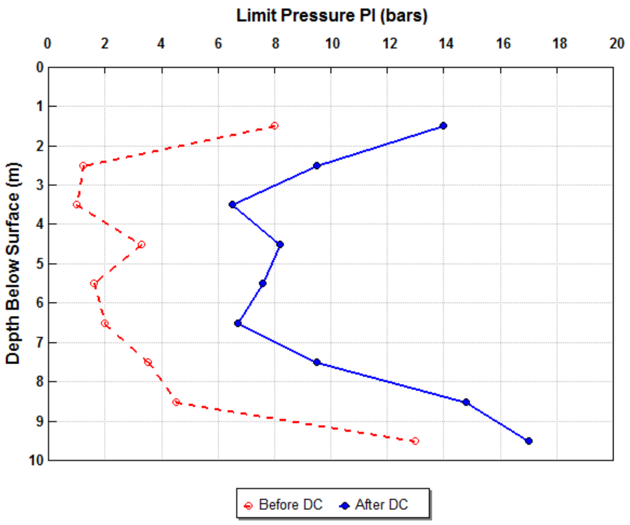


Figure 7 typical PMT test results before and after DC

6.2 *Quality Control for DR*

The quality control for DR and HDR is more stringent as the locations for individual footings were unknown during works. The works procedure specifically developed for DR and HDR is as follow:

- Step 1 - Observational method : Delineate DR treatment area by measuring the penetration of the pounder. Special instrumentation program was installed on the DR rigs to record the GPS coordinates of the impact locations and the total penetration of the pounder. An area mapping is automatically generated and defined for DR treatment.
- Step 2 – Q.C. method : Confirmation by CPT after area mapping. This is to confirm the depth of DR columns required. For depth of compressible Sabkha deposit greater than 5m below the working platform, Heavy Dynamic Replacement (HDR) using 25 tons pounder was used. A 3m surcharge fill was placed after the HDR for a period of 6 weeks to achieve 95% consolidation.
- Step 3 – Analytical method : Confirmation of DR column spacing to ensure adequate soil arching to develop in the engineered fill between the base of the footings and the composite soil-column mass (see Figure 8). The optimised column grid was determined to be 3.89m x 3.82m for columns of minimum 2.2m diameter. The measured column diameter varied between 2.35m to 3.0m. The column size was checked by excavation after dewatering.

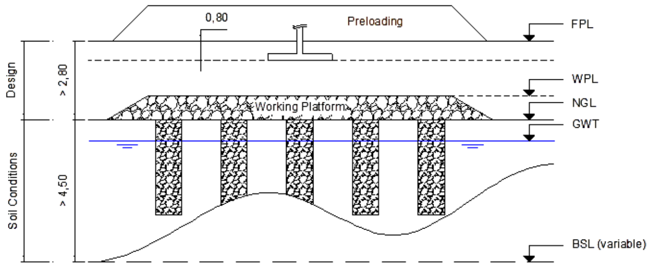


Figure 8 Transition layer above DR columns for soil arching

- Step 4 – Q.C. method : Perform PMT test to verify the mechanical properties of the columns. PMT tests were carried out at the column locations and in-between the columns to measure the mechanical properties of the columns and the surrounding soils. Limit pressure (P_L), creep pressure or yield pressure (P_Y) and pressuremeter modulus (E_M) were measured. Figure 9 shows the P_L values before and after DR treatment inside and inbetween columns.
- Step 5 – Analytical method : Confirmation of non-yielding of soil in-between DR columns upon loading.

Analysis using Finite Element Modelization (FEM) was also performed to verify the stress distribution between soil and DR column and to determine the maximum induced stress in the surrounding soil between the columns. Figure 9 shows the results of such analysis.

Based on the settlement criteria, the induced stress in the surrounding soil was kept below the yield pressure, P_Y where P_Y was taken as $P_L/2$.
To avoid potential creep settlement, a yield pressure P_Y of 90 kN/m² was established as the minimum value to be achieved in the Sabkha deposit in-between the DR columns. With this condition, settlement prediction can be carried out with sufficient accuracy using the design rules given in D-60AN (Soil Sols No. 26, 1975). Figure 10 shows a comparison of P_L values of the Sabkha deposit before and after DR treatment.

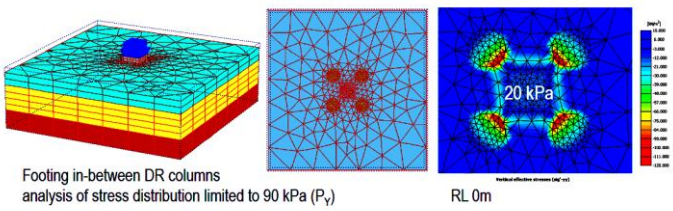


Figure 9 FEM results for stress analysis in-between DR columns

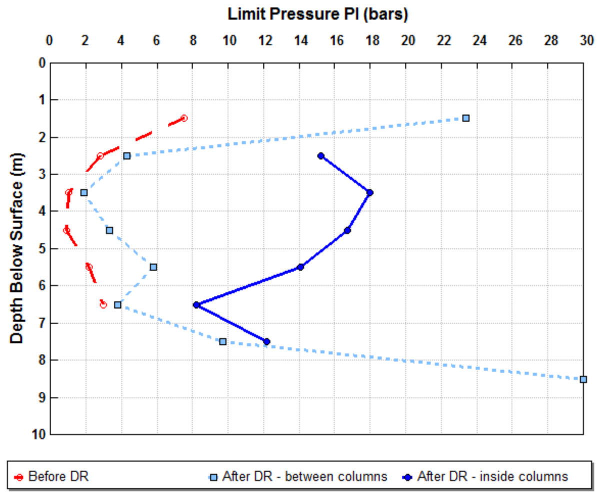
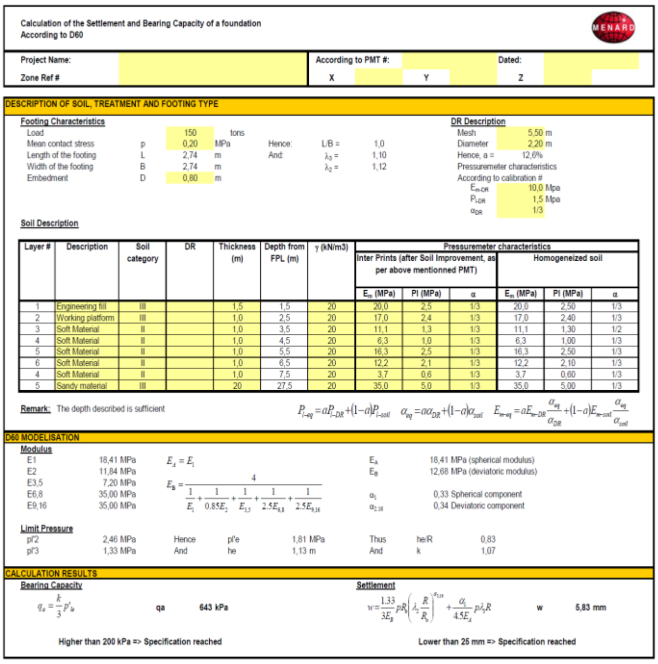


Figure 10 PMT test results before and after DR

- Step 6 – Analytical method : Confirmation of bearing capacity and settlement after treatment. A design spread-sheet was developed for the calculation of bearing capacity and settlement using the results of PMT tests as shown in Figure 11. These parameters constituted the core of the acceptance results analysis.



Calculation of the Settlement and Bearing Capacity of a foundation
According to D60

MENARD

Project Name:		According to PMT #:		Dated:	
Zone Ref #		X	Y	Z	

DESCRIPTION OF SOIL, TREATMENT AND FOOTING TYPE

Footing Characteristics

Load		150	tons			
Mean contact stress	p	0,20	MPa	Hence:	L/B =	1,0
Length of the footing	L	2,74	m	And:	λ_3 =	1,10
Width of the footing	B	2,74	m		λ_2 =	1,12
Embedment	D	0,80	m			

DR Description

Mesh	5,50	m
Diameter	2,20	m
Hence, a =	12,6%	
Pressuremeter characteristics		
According to calibration #		
$E_{m\text{-}DR}$	10,0	Mpa
$P_{l\text{-}DR}$	1,5	Mpa
α_{DR}	1/3	

Soil Description

Layer #	Description	Soil category	DR	Thickness (m)	Depth from FPL (m)	γ (kN/m3)	Pressuremeter characteristics: Inter Prints (after Soil Improvement, as per above mentionned PMT)			Homogeneized soil		
							E_m (MPa)	Pl (MPa)	α	E_m (MPa)	Pl (MPa)	α
1	Engineering fill	III		1,5	1,5	20	20,0	2,5	1/3	20,0	2,50	1/3
2	Working platform	III		1,0	2,5	20	17,0	2,4	1/3	17,0	2,40	1/3
3	Soft Material	II		1,0	3,5	20	11,1	1,3	1/3	11,1	1,30	1/2
4	Soft Material	II		1,0	4,5	20	6,3	1,0	1/3	6,3	1,00	1/3
5	Soft Material	II		1,0	5,5	20	16,3	2,5	1/3	16,3	2,50	1/3
6	Soft Material	II		1,0	6,5	20	12,2	2,1	1/3	12,2	2,10	1/3
4	Soft Material	II		1,0	7,5	20	3,7	0,6	1/3	3,7	0,60	1/3
5	Sandy material	III		20	27,5	20	35,0	5,0	1/3	35,0	5,00	1/3

Remark: The depth described is sufficient

$$P_{l-eq} = aP_{l-DR} + (1-a)P_{l-soil} \quad \alpha_{eq} = a\alpha_{DR} + (1-a)\alpha_{soil} \quad E_{m-eq} = aE_{m-DR}\frac{\alpha_{eq}}{\alpha_{DR}} + (1-a)E_{m-soil}\frac{\alpha_{eq}}{\alpha_{soil}}$$

D60 MODELISATION

Modulus

E1	18,41 MPa		E_A	18,41 MPa (spherical modulus)
E2	11,84 MPa		E_B	12,68 MPa (deviatoric modulus)
E3,5	7,20 MPa			
E6,8	35,00 MPa		α_1	0,33 Spherical component
E9,16	35,00 MPa		$\alpha_{2,16}$	0,34 Deviatoric component

$$E_A = E_1$$

$$E_B = \frac{4}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3,5}} + \frac{1}{2.5E_{6,8}} + \frac{1}{2.5E_{9,16}}}$$

Limit Pressure

pl'2	2,46 MPa	Hence	pl'e	1,81 MPa	Thus	he/R	0,83
pl'3	1,33 MPa	And	he	1,13 m	And	k	1,07

CALCULATION RESULTS

Bearing Capacity

$$q_a = \frac{k}{3}p'_{le}$$

qa 643 kPa

Higher than 200 kPa => Specification reached

Settlement

$$w = \frac{1.33}{3E_B}pR_0\left(\lambda_2\frac{R}{R_0}\right)^{\alpha_{2,16}} + \frac{\alpha_1}{4.5E_A}p\lambda_3R$$

w 5,83 mm

Lower than 25 mm => Specification reached

Figure 11 Design spread-sheet for bearing capacity and settlement according to D-60AN method using PMT results

6.3 *Pressuremeter Tests using a Self-Bored Slotted Tube (Staf)*

As described previously, one of the methods used to verify the design was pressuremeter testing. On this project, inside the DR columns, it was decided to use the Self-boring slotted tube (Staf) method. The idea behind the Staf method is to create a cavity in which the pressuremeter probe is inserted without remoulding or stress relieving the surrounding soils. This is achieved by drill and using a self-boring STAF rotary percussion drilling with slurry circulation and cutting extraction to the maximum depth of the test. The Staf drag bit is them rectracted and raised inside the 63mm cased hole. The 44mm OD pressuremeter probe is then inserted into the cased hole which has a slotted tube at its end and the test can then be performed inside the slotted casing which is then raised to the next test elevation.

Figure 12 STAF drag bit unfolded and folded inside the casing

7 D C-DR IMPROVEMENT CHART

From the results of more than 2,000 PMT tests from this project as well as PMT tests performed near the treatment area, an attempt was made to compile all the data with compaction energy used versus net limit pressure.

Figure 12 attempts to define the limits of improvement for DC and DR based on fines content (FC). The improvement factor (I) ranges between 3 and 8. The lower factor of 3 is mainly obtained in soils of high fine contents (FC = 50%). SI is the compaction energy specific improvement factor which is the ratio of the improvement factor (I) over the compaction energy used.

Taking an average P_L before treatment as 1 bar (100 kPa) and based on the required bearing capacity with a factor of safety of 3, the required P_L after treatment is 7.5 bars (750 kPa). With fines content of 10% and an improvement factor of 8, DC is deemed applicable. However, with fines content of 50% and an improvement factor of 3, the P_L after treatment is less than 7.5 bars. Hence, DR treatment is deemed more appropriate than DC treatment in this case.

8 DYNAMIC SURCHARGE

Upon reaching 78% completion of the ground improvement works, the Final Master Development Plan was issued. In this final plan, 39 buildings of 6-storey high with footprint measuring 25m by 110m had to be included in the ground improvement works. This was not included in the original design concept neither was such a large size for the foundations nor the magnitude of the imposed load. Due to time constraints, the original design concept was modified to allow a 6m surcharge for these buildings. However, after completion of surcharge for 3 buildings, it was realized that the access ramps for the earthmoving trucks made this scheme not feasible for the remaining buildings. The technique of dynamic surcharge was proposed instead of the static fill surcharge.

Dynamic surcharge has been applied elsewhere e.g. Mobil Oil tank farm in Jurong, Singapore (Yee *et al.* 1997). A theoretical approach to dynamic surcharge consists of analyzing the pore pressure behaviour during consolidation in a similar manner to that of a static fill surcharge. It is assumed that the dynamic impacts generate an excess porewater pressure at least equals to the pore pressure generated by the surcharge embankment load. The theoretical approach is described in Varaksin *et al.* (2009).

Fig. 13 shows the settlement induced by the dynamic surcharge. Settlement of 4 cm was recorded under a 3 m surcharge over a period of about a week. With two phases of heavy dynamic impacts on top of the 3 m surcharge, an additional 12 cm settlement was induced. Figure 14 shows the dynamic surcharge being carried out on top of the 3m fill surcharge.

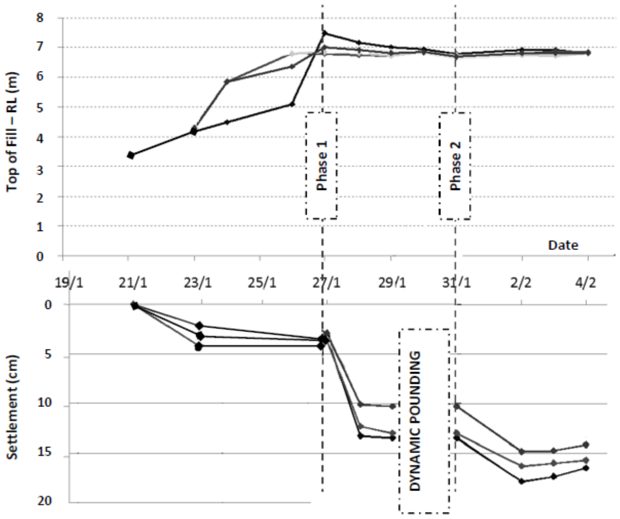


Figure 13 Settlement due to dynamic surcharge

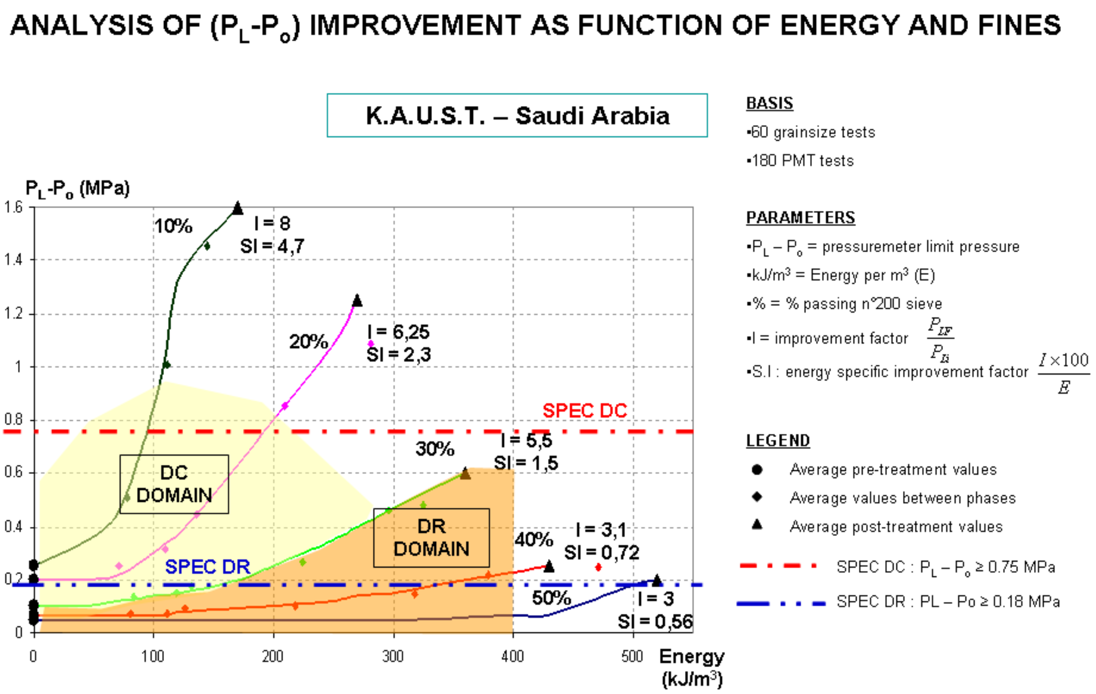


Figure 12 DC-DR improvement chart

Figure 14 Dynamic surcharge on top of 3m fill surcharge

Even though the limit pressures P_L at the end of the 3m surcharge were not tested, after the dynamic surcharge and subsequent removal of the surcharge fill, the P_L values increased from an average of 80 kPa before treatment to 190 kPa after DR treatment and 750 kPa after dynamic surcharge. These combined effects led to a much better performance than initially expected. However, this phenomenon needs to be verified with further case studies on different ground conditions elsewhere.

9 CONCLUSION

The KAUST project is a fast-track project where the design, planning and construction were undertaken concurrently. This mega size project entailed site preparation involving ground improvement for structures on non-defined locations over an area of 2,600,000 m^2. Due to the heterogeneous ground conditions and unknown locations for footings, a design concept for the ground improvement works was formulated. The extent and the degree of improvement required were based on field observation method and re-confirmed by Q.C.based on CPT and PMT. Ground improvement was carried out using a combination of DC, DR, HDR and fill surcharge. Detailed quality control on the works using q_c and R_f values from CPT and P_L and E_M values from PMT was carried out. Due to unforeseen circumstances and project constraints, dynamic surcharge was tried out on this project to supplement the conventional static fill surcharge. The results obtained were better than expected and this warrants further investigation into this new phenomenon. Figure 15 shows the area extent covered by DC/DR treatment on 2,600,000 m^2 areas completed within the stipulated time period in spite of the challenges imposed during the works.

Figure 15 DC/DR works on 2,600,000 m^2 area at KAUST

REFERENCES

D.60.AN. 1972 *Interpretation and Application of Pressuremeter Tests Results to Foundation Design*. Sol Soils No. 26.

Kenny Yee, Jean Jacque Richard, Serge Varaksin & Goh Tow Peng (1997). "Behavior of Tank Foundation on Improved Ground at Jurong, Singapore" *Proceedings of the 30th Year Anniversary Symposium of the Southeast Asian Geotechnical Society on Deep Foundations, Excavations, Ground Improvements & Tunneling*, Bangkok, Thailand, November 1997

Serge Varaksin, Kenny Yee & Wong Leong Toh (2009) “Formulation of a Concept and Realistic Soil Parameters for the Foundations of a Randomly Located Structures on a Mega Scaled Project” *Proceedings of the International Symposium on Ground Improvement Technologies and Case Histories (ISGI09)*, Singapore, 9 – 11 December 2009 – Keynote Paper.

G.Arsonnet et al (2005) “Pressuremeter tests inside a self-bored slotted tube (STAF)” – *Proceedings of ISP5 – Pressio 2005*, Marne-La-Vallee, France, 22-24 August 2005